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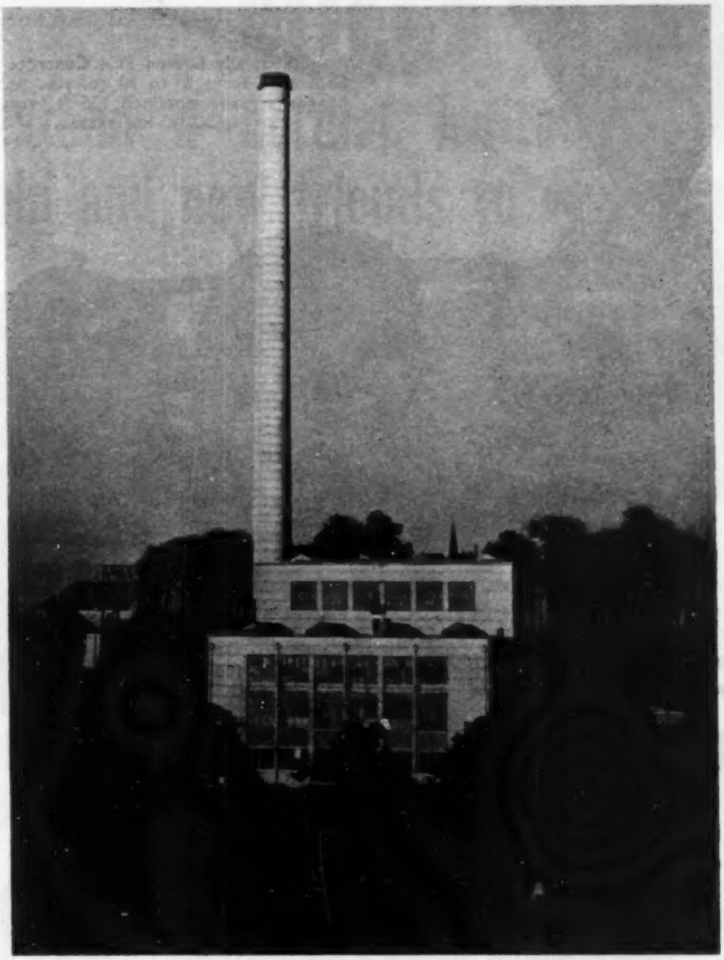

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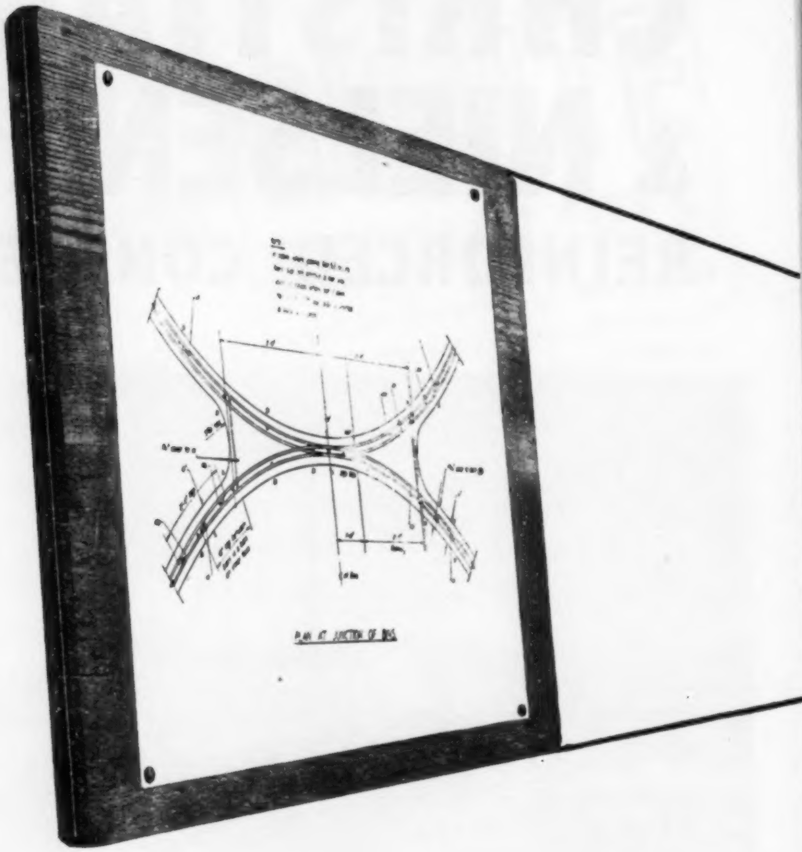

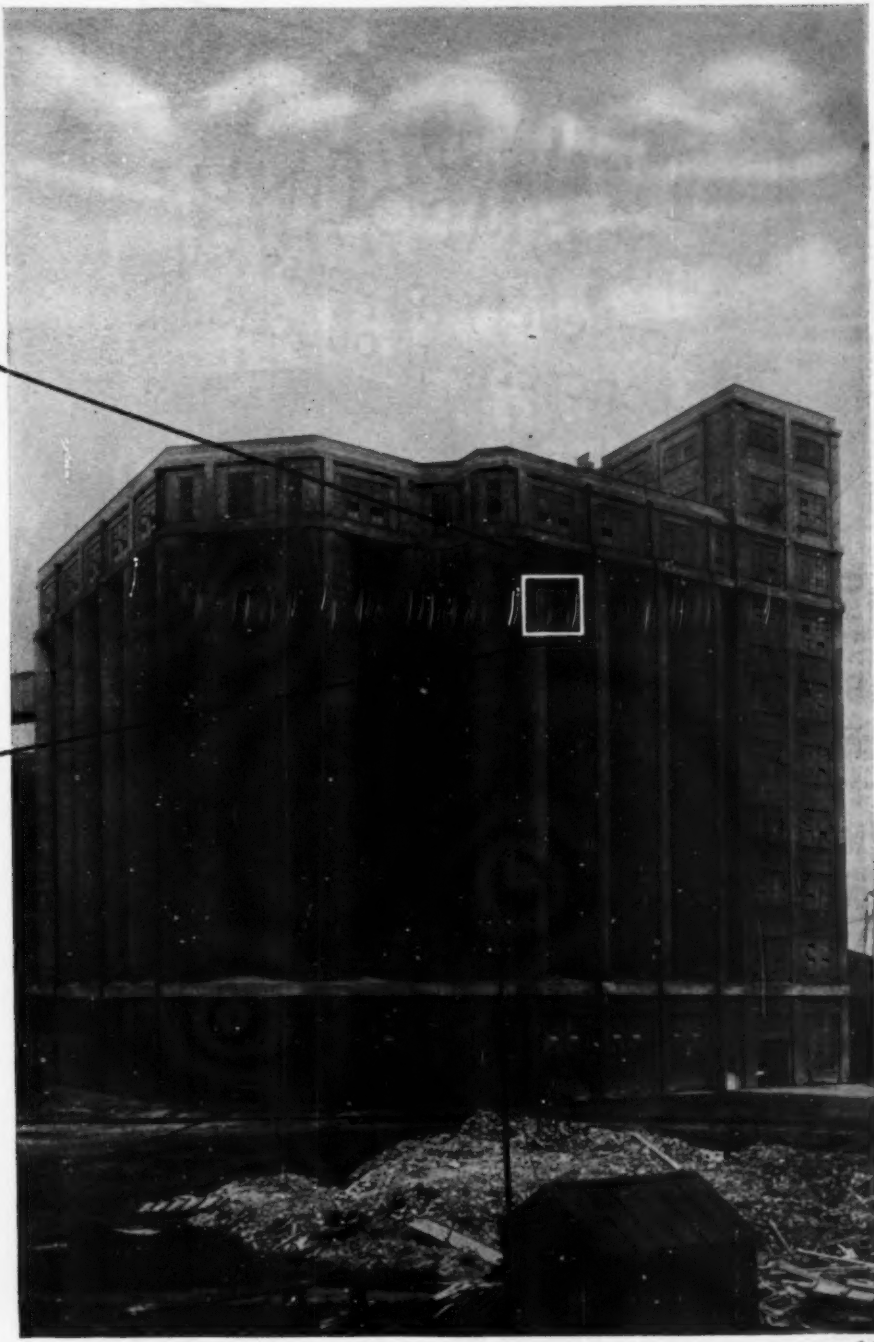

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CONCRETE AND CONSTRUCTIONAL ENGINEERING

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EDITORIAL NOTES

New Forms of Structure.

IT is unfortunate that words and phrases which are unintelligible to those who do not know what is in the mind of the writer, and who can apply to words only their ordinary meaning, should be used in discussions of the æsthetics of structures. In a recent work a reinforced concrete frame building is said to "provide the basis for an architectural expression true to the nature of the materials of which it is composed"; this seems to be meaningless when it is obvious that the same elevation could have been built with a steel frame. A block of residential flats with balconies is claimed to "express its nature as a series of superimposed dwellings," while another building without balconies and used as a hostel is said to express through its structure the individual rather than the dwelling. Under another photograph we are told that the columns are "expressed by means of rainwater pipes placed where the [invisible] columns occur and picked out in colour." It may be that the architect thought that others would know that rainwater pipes indicate the positions of columns, but we cannot see that he had any justification for assuming that ordinary people are gifted with such intuition.

The quotations are from a book * just published in the United States, and are a small blemish in an important work. The publication in a sumptuous volume of more than two hundred photographs of modern structures, many of which are in reinforced concrete, is a notable event. Indeed this book, with its large and beautifully-printed pages, could almost be compared with some of the volumes published in the last century illustrating the works of great artists and architects, except that slickness of presentation has replaced the dignity of the older volumes. Its appearance is, however, perhaps intended to be in accord with the purpose of the book, which is to illustrate new and often unusual forms of construction and to point out their advantages compared with older methods. Although many of the illustrations will be familiar to those who see the technical journals, the collection of so many in one volume is an impressive indication of the increasing use of new forms of construction throughout the world, and the author's notes on the reasons for the development of these forms are valuable. It is right that architects and building owners should know of what is now possible, and how engineers (few of whom are British, by the way) have in recent years made available new shapes and forms which can be used in solving problems of planning unobstructed floor space or of producing elevations and roof shapes

* "Contemporary Structure in Architecture." By Leonard Michaels. (London: Chapman & Hall, Ltd. Price 68s.)

different from anything that was economically possible only a decade or two ago. This excellent book shows how new forms are being used by engineers and architects, some of whom are, perhaps naturally, attempting to make them fashionable as well as to demonstrate their soundness and economy. Whether all of these forms will be accepted by this or a future generation as having artistic merit only time will tell. It may well be that their international origin is not in their favour, for the architectural styles that have survived and are still copied were conceived and perfected in one country only.

The author does not dwell on the æsthetic merits of the structures illustrated. The purpose of the work is to show the new forms that are available, and the reader is generally left to form his own judgment of their value from the artistic point of view. Wisely, also, it is not predicted to what extent these forms will be used in the future. A notable feature of nearly all the buildings illustrated is the absence of ornament, and dependence for effect on line and the massing of solids and voids. This may lead to cheapness and may be symbolic of the hard times in which we live, but it is far from certain that it is a satisfactory substitute for decoration and ornament. The skill of the carpenter on shuttering and of the punner of concrete in producing true lines and surfaces is not a substitute for the skill of the carver and sculptor. The most that can be said of a clean concrete surface and good proportions is that they do not offend the eye, whereas good decoration can give pleasure. It may be, however, that a new generation may have a quite different view of æsthetics than that which is commonly held to-day. It may be that in future decoration and ornament on structures will be looked upon as being as old-fashioned as a Victorian drawing room, and that a more intellectual people will judge the æsthetics of a structure by the ease (or difficulty) with which an observer can guess that rainwater pipes stand for columns, balconies for dwellings, no balconies for individuals, and so on.

Abstractions such as truth, goodness, justice, and their opposites, mean different things to different people. They have been the subject of argument for thousands of years, and while human beings retain the power of independent thought such words will not have the same meaning to all people. The proviso that the power of independent thought be retained is important, for we have seen how education, propaganda, and reiteration can make whole nations (or the majority of their populations) believe that truth, goodness, and justice mean quite the opposite of what they are believed to mean by nations who have not been subjected to such intensive teaching. Beauty is another abstraction which always has had and always will have different meanings to different people, because different people have different tastes. Ideas of beauty are, however, often influenced by a few vociferous critics whose views become accepted as fashionable, and those who do not agree with them are looked upon as unreliable deviationists in much the same way as are those who do not entirely agree with the views of the dictator of the moment. Why must a building be expected to "express" anything at all? Why is it not sufficient that it give pleasure to those who see it? We may assume that a new structure serves a certain purpose because buildings having the same general appearance have been commonly used for that purpose in the past, but this is a result of association of ideas. A building can express nothing.

An Analysis of Flat Slabs.

By Dr. A. M. HAAS [The Hague, Holland].

[The following is abstracted, by arrangement, from the published works * of the author.]

The design of flat slabs (or "beamless" or "mushroom" floors) can be based on the recommendations of codes of practice (or the regulations of governing authorities) or on theoretical analysis. The two methods may not always give the same results for the reasons discussed in the following, in which is described also an improved theory.

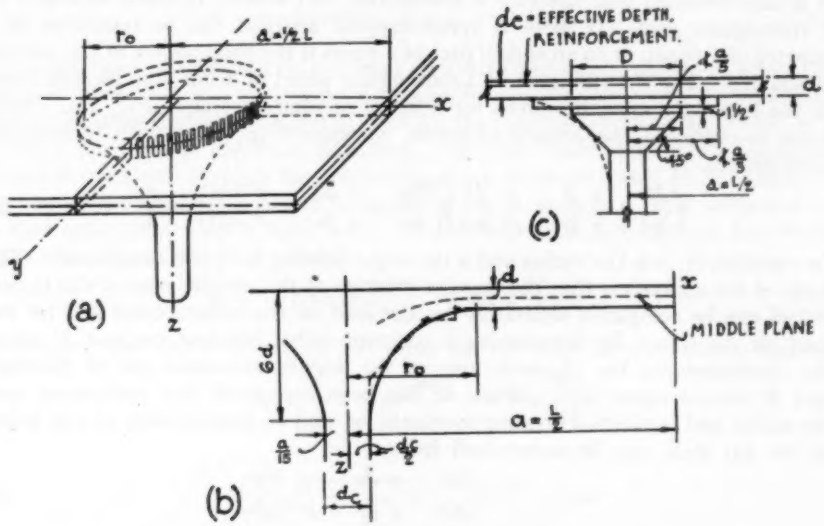


Fig. 1.

The theoretical analysis of flat slabs is generally based on the differential equation

$$\frac{\partial^4 u}{\partial x^4} + 2\frac{\partial^4 u}{\partial x^2\,\partial y^2} + \frac{\partial^4 u}{\partial y^4} = \frac{w}{N} \quad . \quad . \quad . \quad . \quad (1)$$

which is the general equation relating the uniformly-distributed load w on a plate to the deflection u, and the derivation of which is given in textbooks dealing with the theory of plates. In this equation x and y are rectangular co-ordinates and N is the flexural rigidity $\dfrac{Ed^3}{12(1 - v^2)}$, in which E is the elastic modulus and v is Poisson's ratio. It is assumed in the derivation that the thickness d of the plate is constant, which is not the case at the column head of a flat slab. One of the

* "Ontwerp en Berekening van Paddestoelvloeren." (The Hague: Martinus Nijhoff. 1949. With a summary in English. Price 13 guilders.)
"The Calculation of Flat Slab Floors." Final Report of the Third Congress of the International Association for Bridge and Structural Engineering, 1948; page 535.
"Conception et calculs des planchers à dalles champignons." (Paris: Editions S.I.D.E.S.T. 1950.)

difficulties of the problem is to allow for the column head, and in most analyses equation (1) is solved for a slab of uniform thickness and the influence of the column head is included afterwards by means of an approximation.

In the analysis developed, the column head is included at the beginning and for convenience of calculation the column head is assumed to be a hyperboloid (*Figs. 1a* and *b*). In *Fig. 1c* the assumed shape is superimposed on a column head of the shape specified in the British Standard Code (CP 114.102—1950) for flat slabs. The analysis comprises two parts, namely, the column head and the slab. If axial symmetry of shape and load is assumed and polar co-ordinates are used, there is only one variable for the column head and two (the radius and angle) for the slab, and a solution can be obtained. To conform with the general equation, it is also assumed that the slab is continuous over several columns arranged on a rectangular grid. If there is symmetry the analysis can be restricted to a quarter of a panel, or to an eighth part of a panel if the columns are at the corners of squares. For the slab part of a rectangular panel of a flat slab the condition at the boundary must be solved for a figure of complex shape (*Fig. 2a*). With polar co-ordinates the general equation, corresponding to (1) with rectangular co-ordinates, is

$$\left(\frac{\partial^2}{\partial r^2}+\frac{1}{r}\cdot\frac{\partial}{\partial r}+\frac{1}{r^2}\cdot\frac{\partial^2}{\partial \alpha^2}\right)\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\cdot\frac{\partial u}{\partial r}+\frac{1}{r^2}\cdot\frac{\partial^2 u}{\partial \alpha^2}\right)=\frac{w}{N} \quad . \quad . \quad (2)$$

In equation (2), r is the radius and α the angle defining the point considered. The basis of the analysis is that the angular rotation at the circular edge of the hyperboloid can be computed separately for the load on the column-head and for the load on the slab. By introducing a constant radial bending moment M along the circumference, the algebraic sum of the angular rotations due to the load and M should equal zero. When M has been computed, the deflections, and the radial and tangential bending moments (m_r and m_t respectively) at any point of the flat slab, can be ascertained from

$$m_r=-N\left(\frac{\partial u}{\partial r^2}+\frac{v}{r}\cdot\frac{\partial u}{\partial r}+\frac{v}{r^2}\cdot\frac{\partial^2 u}{\partial \alpha^2}\right) \quad . \quad . \quad . \quad (3)$$

$$m_t=-N\left(v\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\cdot\frac{\partial u}{\partial r}+\frac{\partial^2 u}{r^2\partial \alpha^2}\right) \quad . \quad . \quad . \quad (4)$$

In the following, the basic assumptions and formulæ and the more important deductions of the analysis are given. Intermediate steps, which are given in Dr. Haas's book, are omitted.

Analysis of Column Head.

For the column-head, d in the expression for N is a variable d_z and equals Cr^{-1}. Also C equals xz. At the edge of the column-head, $z = d$ (the thickness of the slab) and $x = r_0$ (the radius of the column-head). Assuming $r_0 = 0{\cdot}4a$ (a being one-half of the span of the panel), $C = 0{\cdot}4ad$. Therefore $d_z = \frac{0{\cdot}4ad}{r}$. The slope ϕ and deflection u at the edge of the column-head ($r = r_0$), assuming v is one-sixth, due to the effects enumerated as (i), (ii), and (iii) in the following, can be calculated. To facilitate the calculation, $\frac{K}{r^3}$ can be substituted for N if

$K = \frac{(0{\cdot}4ad)^3E}{12(1 - v^2)}$, but formulæ (5) and (5*a*) to (5*d*) are given in terms of N.

(i) A uniformly-distributed load on the column-head:

$$\phi = 0{\cdot}00062\frac{wa^3}{N}\,;\quad u = 0{\cdot}000117\frac{wa^4}{N} \quad . \quad . \quad . \quad (5a)$$

(ii) The shearing force (the load from the slab) around the edge of the column-head:

$$\phi = 0{\cdot}013856\frac{wa^3}{N}\,;\quad u = 0{\cdot}002123\frac{wa^4}{N} \quad . \quad . \quad . \quad (5b)$$

(iii) A moment M acting at the edge of the column-head:

$$\phi = -\,0{\cdot}1099\frac{Ma}{N}\,;\quad u = 0{\cdot}0098\frac{Ma^2}{N} \quad . \quad . \quad . \quad (5c)$$

Variations of Poisson's ratio v from 0 to one-third can be shown to affect the slope by about 18 per cent. and the deflection by about 12 per cent. Deformation due to shearing stresses is neglected since for the column-head it accounts for only about 1 per cent. of the total deflection. The dispersion of the shearing forces (vertical reactions) through the column-head can be investigated by Boussinesq's theory as modified by Bÿlaard to apply to the stresses in a plate subjected to a load concentrated on a small area. At the edge of a column-head for which $r_0 = 0{\cdot}4a$, it can be shown that

$$\phi = -\,0{\cdot}00020\frac{wa^3}{N}\,;\quad m_r = +\,0{\cdot}54wa^2 \quad . \quad . \quad . \quad (5d)$$

Combining the slopes in (5*a*) to (5*d*) gives the net slope at the column-head as

$$\phi_{r=r_0} = 0{\cdot}01428\frac{wa^3}{N} + 0{\cdot}1099\frac{Ma}{N} \quad . \quad . \quad . \quad (5)$$

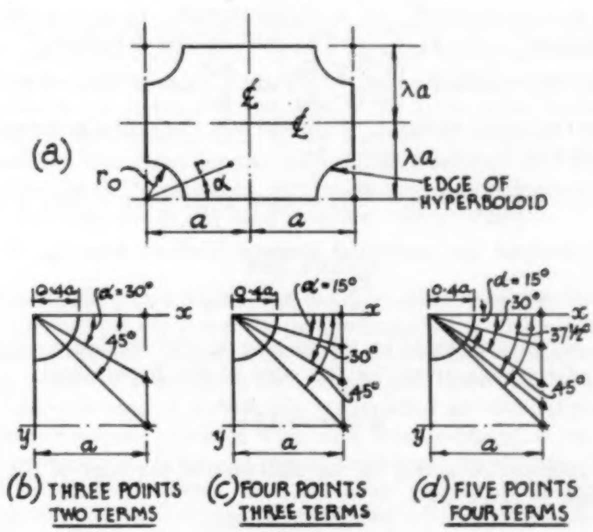


Fig. 2.

Analysis of the Slab.

The deflection u in equation (2) can be divided into two parts u_0 and u_1 such that $u = u_0 + u_1$, in which u_0 is a solution which satisfies (2) and u_1 is the solution which satisfies (2) when it is equated to zero. Text-books on elasticity give expressions for u_1 in terms of functions of r and α (*Fig. 2a*) involving a complex expression which is not reproduced here but is given in Dr. Haas's book. The solution of the differential equation is given as a series, the number of terms in which corresponds to the number of points chosen on the boundary of the quarter of the panel of the slab shown in (*b*), (*c*), and (*d*) in *Fig.* 2. At these points, the number of which should be as small as possible for the solution of practical problems, the boundary conditions have to be satisfied. The series is in general convergent, but for a small area near the middle of the panel convergence is confined to a limited number of terms. For the midpoint of the panel less than four terms of the series should be used. In the square panel in *Fig. 2b* only three points $\left(\alpha = 0,\ r = a\ ;\ \alpha = 30 \text{ deg.},\ r = \dfrac{2a}{\sqrt{3}}\ ;\ \text{and } \alpha = 45 \text{ deg.},\ r = a\sqrt{2}\right)$ are shown. Therefore two terms only of the series are used and give the expression

$$u = \frac{w}{N}\left[\frac{r^4}{64} + A_0 + B_0 \log_e \frac{r}{r_0} + C_0 r^2 + D_0 r^2 \log_e \frac{r}{r_0} + (A_1 r^4 + C_1 r^6) \cos 4\alpha \right.$$
$$\left. + (A_2 r^8 + C_2 r^{10}) \cos 8\alpha \right] \quad . \quad . \quad (6)$$

The constants A and C relate to the centre-lines of the panels of the slab (*Fig.* 2) and the constants B and D to the edge of the hyperboloid ($r = r_0$). A_0 and C_0 can be expressed in terms of B_0 and D_0. For a uniformly-distributed load, for each of the three points defined in the foregoing and shown in *Fig. 2b*, numerical values of these factors can be derived in terms of a in the following order: $D_0 = -0{\cdot}159310a^2$, $B_0 = +0{\cdot}001672a^4$, $C_0 = +0{\cdot}206367a^2$, and $A_0 = -0{\cdot}033419a^4$. A_1, A_2, C_1, and C_2 can also be determined.

From (6), expressions for $\dfrac{\partial u}{\partial r}$, $\dfrac{\partial^2 u}{\partial r^2}$, $\dfrac{\partial u}{\partial \alpha}$, and $\dfrac{\partial u^2}{\partial \alpha^2}$ can be derived for substitution in (3) to give the radial moment. Consequently the radial moment at the edge of the hyperboloid ($r = r_0$) is given by

$$m_r = -w\left[\frac{r^2}{16}(3 + v) - \frac{B_0}{r^2}(1 - v) + 2C_0(1 + v) + D_0(3 + v)\right] \quad . \quad (7)$$

By a similar analysis the tangential moment [derived from (4)] is given by

$$m_t = -w\left[\frac{r^2}{16}(3v + 1) + \frac{B_0}{r^2}(1 - v) + 2C_0(1 + v) + D_0(3v + 1)\right] \quad . \quad (8)$$

Substitution of the constants in (6) to give u, differentiation to give ϕ, and substitution of $r = 0{\cdot}4a$ gives, for the edge of the hyperboloid,

$$\phi = 0{\cdot}10955\frac{wa^3}{N} \quad . \quad . \quad . \quad . \quad . \quad (9a)$$

For the moment M acting on the slab around the edge of the hyperboloid the mean slope is given by

$$\phi = -0{\cdot}35704\frac{Ma}{N} \quad . \quad . \quad . \quad . \quad . \quad (9b)$$

Combining (9*a*) and (9*b*) gives the net slope as

$$0{\cdot}10955\frac{wa^3}{N} - 0{\cdot}35704\frac{Ma}{N} \quad . \quad . \quad . \quad . \quad (9)$$

Combination of Analyses of Column-head and Slab.

Equating (5) to (9) shows that the moment M around the edge of the hyperboloid ($r = r_0 = 0{\cdot}4a$) is $0{\cdot}2040wa^2$. Combining the data so far established, the theoretical radial and tangential bending moments due to a uniformly-distributed load at the critical sections can be shown to be

	m_r	m_t
Around the edge of the hyperboloid ($r=r_0=0{\cdot}4a$) :	$-\ 0{\cdot}204wa^2$	$-\ 0{\cdot}1231wa^2$
At midspan of a column strip ($r=a, \alpha=0$) :	$+\ 0{\cdot}1352wa^2$	$-\ 0{\cdot}0649wa^2$
,, ,, ,, middle ,, $\left(r=\frac{a}{\sqrt{2}}, \alpha=90 \text{ deg.}\right)$:	$+\ 0{\cdot}1019wa^2$	$+\ 0{\cdot}1004wa^2$

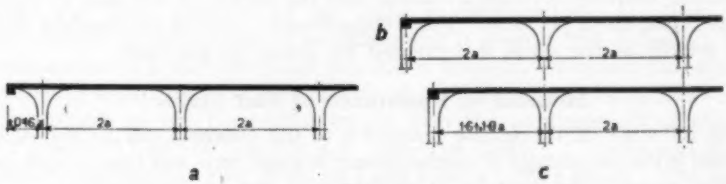


Fig. 3.

Calculation shows that the greatest stresses do not occur around the edge of the hyperboloid ($r = 0{\cdot}4a$) but at a circle of smaller radius, about $r = \frac{a}{3}$, where the radial bending moment is $0{\cdot}3324wa^2$ and the consequent stress in the deeper section is about 13 per cent. higher than at the edge of the hyperboloid.

It should be kept in mind that the foregoing numerical values apply to an example in which the radius r of the edge of the hyperboloid is $0{\cdot}4a$, the hyperboloid replacing the column-head for the purpose of analysis. Comparing the bending moments calculated for the shape of column-head used in practice with those calculated for the hyperboloid, with reasonable approximations the difference is from 13 per cent. to 16 per cent. for the critical sections.

End Panels.

When considering end panels, the bending of the external columns must be taken into account and consequently the investigation of the slab in an end panel cannot be limited to one-quarter of the slab but must be extended to two adjacent quarters, an extension that makes the computation more elaborate but not more difficult. The provision of end panels constructed as internal panels (*Fig.* 3*b*) with external columns, provided with half column-heads, is an uneconomical method of construction. The end panels must resist 20 per cent. to 30 per cent. more positive bending moment than the inner panels at the same permissible stresses, and the external columns have to resist a considerable bending moment. One way to eliminate the influence of the edges is to reduce the span so that

the bending moments become about the same as those in the inner panels, that is by reducing the distance between the columns to about 80 per cent. of that between inner columns (*Fig. 3c*). This method results in unequal spacing of the columns, which offsets the advantage of the constant thickness of the slab and nearly equal reinforcement. A better arrangement is to provide the external columns with a complete column-head and suitable strip of slab as in *Fig. 3a*. By this means the first row of columns is set back from the wall, and the ordinary distance between the columns is then maintained throughout. The continuous slab ends on a straight line which nearly coincides with the line along which there is no bending moment in the continuous slab.

Non-continuous Load.

The effect of non-continuous uniformly-distributed load can be investigated by considering loaded strips extending from column to column alternating with unloaded areas of the same width. In this case, the positive bending moment at a section midway between a column and the middle of the slab is increased by 41 per cent. to 50 per cent. For "chess-board" loading the bending moment at the middle of the panel is increased by about 35 per cent.

Moment of Resistance of Flat Slabs.

The influence of the tensile resistance of the concrete can be shown to be important if the percentage of reinforcement is small (say, less than 0·5 per cent.), and often exceeds the moment of resistance of the reinforcement.

Coefficients in codes are chiefly based on data obtained by tests in which the elongation of the reinforcement is measured. If the slab is not cracked, the moment of resistance is a combination of that due to the tensile stress in the reinforcement and that due to the tensile stress in the concrete. For a slab with less than 0·5 per cent. of reinforcement the bending moment that can be resisted when the slab is not cracked is considerably more than the calculated resistance assuming the slab to be cracked. For slabs with larger percentages of reinforcement this may not be true as it depends on the tensile strength of the concrete. With, say, 0·4 per cent. of reinforcement, the moment of resistance due only to the tensile stress in the concrete may be about two-thirds of the total resistance and the total moment of resistance is about 22 per cent. greater than the moment calculated in accordance with the ordinary method that neglects the tensile strength of the concrete. Therefore bending moments deduced from measurements of the elongation of the steel will be very small for slabs with little reinforcement, and will account for the small positive bending-moment coefficients, especially at the middle of the panel, specified in some codes.

Comparison of Theoretical Results and Requirements of Codes.

The theory described makes it possible to determine the bending moment at any point in the slab. Generally the greatest bending moment occurring at a point is assumed also to be the bending moment on the whole strip of the slab. A slab loaded to failure acts, because of plasticity, as a whole and the bending moments correspond more closely to those of the codes than is the case when the working load operates. At working loads there is a great difference between the

theoretical and specified negative bending moments at the column-head. Earlier theoretical values are low, and are only about half those obtained by more exact analyses. The code of the American Concrete Institute, as revised in 1947, complies more closely with theory. A clause is now included to the effect that, for calculating the negative moment in a flat slab, only three-quarters of the width of the strip may be allowed when determining the compressive stresses in the concrete, and, if a drop is provided, only three-quarters of the width of the drop may be allowed. Large compressive stresses will occur over the column and, to render these permissible, the compressive strength of the concrete must be high. Consequently the tensile strength of the concrete will be high, and as a result the lateral distribution in the slab will be good and a wide distribution of load may be expected should the reinforcement be overstressed. The reduced width of slab referred to in the foregoing does not apply when calculating the quantity of reinforcement.

As already explained, in parts of a flat slab where there is little reinforcement, such as the middle of the panel, the difference between the theoretical bending moments and those required by the codes is considerable. At other critical sections, where the percentage of reinforcement is greater, the difference is less. The arch-action of a flat-slab panel also has a tendency to reduce the difference as regards the negative bending moment in the middle strips between the columns.

Comparison of the theoretical values with the requirements of the latest British Standard Code of Practice [CP. 114—102 (1950)], " Floors and Roofs of Flat Slab Construction ", are given in *Table* I for a square panel. The assumed shapes of the column-heads are given in *Fig.* 4. In the theoretical analysis Poisson's ratio is assumed to be 0·167

Of the four bending moments in *Table* I, three agree fairly well, but the negative bending moments in the column-strip do not agree. However, a comparison between the requirements of codes and theoretical results has only a relative significance. The codes give values which are compromises to allow for the width of a strip, whereas the theoretical bending moments relate to a point and change from point to point. Also, if a slab is loaded to failure the slab will

TABLE I.—COMPARISON OF BENDING MOMENTS.

Bending moment	Strip	British Standard Code		Theoretical bending moment. $r = 0{\cdot}225a$ $L = 2a$
		Formula for total bending moment on strip (with drops)	Bending moment per foot width of slab.* $D = 0{\cdot}225L$ Width of drop $= 0{\cdot}4L$	
Negative	Column	$-0{\cdot}046wL\left(L-\frac{2D}{3}\right)^2$	$-0{\cdot}077wL^2$	$-0{\cdot}140wL^2$
	Middle	$-0{\cdot}016wL\left(L-\frac{2D}{3}\right)^2$	$-0{\cdot}023wL^2$	$-0{\cdot}016wL^2$
Positive	Column	$+0{\cdot}022wL\left(L-\frac{2D}{3}\right)^2$	$+0{\cdot}034wL^2$	$+0{\cdot}034wL^2$
	Middle	$+0{\cdot}016wL\left(L-\frac{2D}{3}\right)^2$	$+0{\cdot}023wL^2$	$+0{\cdot}026wL^2$

* The bending moments per foot width of slab are adjusted to allow for the width of the column-strip being equal to the width of the drop.

act as a whole, because of the effects of plasticity, and then the bending moments correspond more closely to the values in the code than is the case at working loads. Taking these two considerations together, it appears reasonable to assume a value which is less than the maximum bending moment at any point of the strip considered. For the bending moment at the centre of the panel there is another factor to consider as, in this under-reinforced part of the slab, there is additional resistance to bending due to the tensile resistance of the concrete. Therefore the value of $0{\cdot}023wL^2$ in the code compared with $0{\cdot}026wL^2$ is still too great.

The revision of the rules for flat slabs of the American Concrete Institute, already mentioned, is an improvement on previous practice on which the British code is based. There is a reduction of the total of the positive and negative

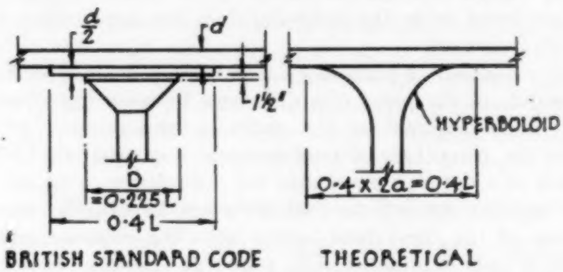


Fig. 4.

bending moments, which total is 90 per cent. of the total in accordance with the British rules, and the distribution is better. The negative bending moment on the middle-strip is reduced but is more in conformity with the theory. The negative bending moment at the column-head agrees rather well with theory if the additional requirement, that for a section through a drop three-quarters of the width of the drop should be assumed to be the width of the section, is also considered. This results in bending moments of $-\ 0{\cdot}13wL^2$ per foot width for the compressive resistance and $-\ 0{\cdot}065wL^2$ per foot width for the tensile resistance. This comparison is for a flat-slab in which the width of the column-head is $0{\cdot}225L$ and the width of the drop is $0{\cdot}4L$ (*Fig.* 4). The conclusions, however, in general hold true and should lead to rules which are more in conformity with a reliable theory.

Tests.

The necessity of more experimental research on flat slabs is self-evident. Although much has been done by test loads and the measurements of strain, the factors which influence the results are so varying that investigations by which the behaviour of flat slabs can be determined accurately are essential. These factors can be divided into two groups, in one of which are those factors that result from the assumptions incorporated in the theoretical analyses of a plate [equation (1)]. In the theory a thin plate is imagined while in practice thick slabs occur. It is also assumed that Hooke's law applies and that the stresses are proportional to the distance from the neutral axis. In the other group are consequences of the fact that reinforced concrete is not a homogeneous material.

Research in which the factors relating to non-homogeneity can be neglected

has been started by the Central National Council for Applied Scientific Research in the Netherlands on a steel model of about one-seventh of actual size. The model comprises three rows each containing five panels. The stresses and deflections are measured by the latest types of apparatus. Although it will be necessarily a long time before definite conclusions are reached, the first results indicate that the measured deflections are about 15 per cent. less than the theoretical deflections.

Strengthening a Viaduct.

HURSTBOURNE viaduct, on the main-line railway between Basingstoke and Andover, was built in 1854. The structure has nine semi-circular brick arches of 46 ft. span supported on brick piers, the greatest height of the rails being about 65 ft. above the ground. The ballast and double line of rails were supported on five longitudinal brick jack-arches spanning transversely and springing from spandrel walls carried on the main arches and piers. The piers, which are founded on chalk, and the main arches show no defects, but the spandrel walls and jack-arches are of light construction and have deteriorated because of the decay of the lime mortar in the joints. To remedy this defect in the past, three tiers of transverse tie-rods had been inserted at the piers, and one tier over the main arches. Chalk had been deposited over the jack-arches to increase the inertia of the structure, and this raised the rails 2 ft. As the defect still persists, works to strengthen the structure are now proceeding (*Figs.* 1 and 2).

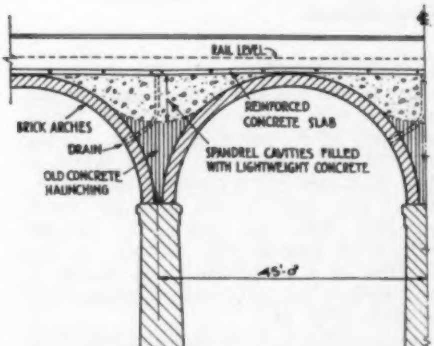


Fig. 1.—Longitudinal Section.

Replacement of the walls and jack-arches by reinforced or plain concrete would not be a remedy as tests indicate that additional weight at a high level is undesirable and may tend to cause side sway. The strengthening measures being taken (*Fig.* 1) therefore include the removal of the chalk filling, demolition of the jack-arches, and filling with lightweight concrete over the extrados of the main arches up to the underside of a new reinforced concrete slab which extends the entire length and width of the viaduct. As a result of these works the weight of the structure is not altered, but the centre of gravity of the deck is lowered and, since the rails are lowered about 2 ft., the centre of gravity of the moving load is also lowered.

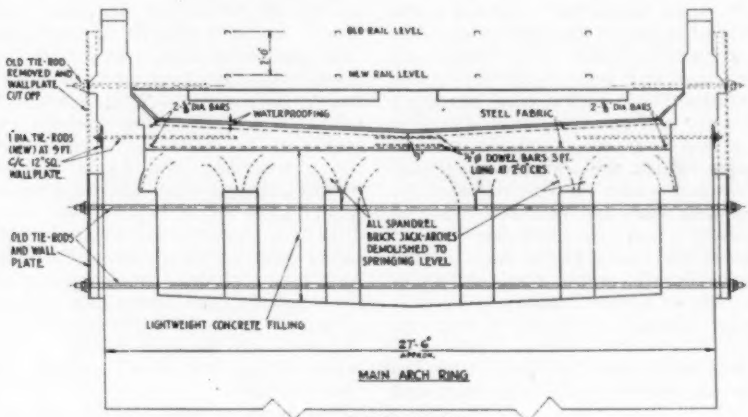
The work is being carried out in two sections by adopting single-line working. To maintain the stability of the structure while one-half of the width of the viaduct only is subjected to live load, it is necessary to fill the spandrels without removing a large quantity of chalk filling. Timbered shafts 15 ft. by 5 ft. are sunk over each pier under the up-line, and the two jack-arches under one track and the one under the six-foot way are broken through. The lightweight concrete, which is mixed on the viaduct, is discharged directly from the mixer into the cavities.

The lightweight concrete, of which there is 800 cu. yd., is composed of foamed slag and ordinary Portland cement in the proportions of 1 cwt. of cement, 4 cu. ft. of foamed slag from $\frac{1}{8}$ in. down, and 4 cu. ft. of foamed slag $\frac{1}{8}$ in. to $\frac{1}{2}$ in. These quantities of materials require from 9 to $9\frac{1}{2}$ gallons of water to give a fairly dry mixture, the weight of which when set is

about 90 lb. per cubic foot. Six-inch cubes, made in metal-faced plywood moulds, have a crushing strength of about 1100 lb. per square inch at seven days. By using concrete of this weight, the total weight of material required to fill the cavities is about equal to the dead load removed.

The next stage is to construct in short lengths a temporary wall in a trench along the longitudinal centre-line of the deck to retain the old filling under the single-line while the filling on the other half of the viaduct is removed. The wall is made of sand-bags filled with dry 1:10 mixture of cement and sand, which is hardened by the effect of the weather but is not so hard that removal upon completion of the work is difficult. As the excavation of the old filling proceeds, the old brick deck over the jack-arches, which were 28 ft. long, is exposed, and the brickwork of the jack-arches and backing, most of which suffers from perished mortar, is demolished and removed, and the void filled with lightweight concrete up to the crown of the main arches. As excavation proceeds, the top tier of old tie-rods is removed, as are also the corresponding parts of the old wall-plates. The two lower tiers of tie-rods are left embedded in the lightweight concrete. The amount of excavation, including that in re-grading the approaches to the viaduct, is about 3000 cu. yd. The ⅜-cu. yd. dragline excavator used is suitable for working in narrow spaces as the jib alone slews and not the cab. Most of the excavated material is dropped over the parapet of the viaduct.

Over the lightweight concrete filling a reinforced slab of 1:2:4 concrete is constructed in short lengths and in a width about half the width of the viaduct. The minimum thickness is 9 in. at the centre of the deck, towards which the top of the slab slopes at 1 in 24 to facilitate drainage. The slab is waterproofed by a liquid bituminous compound and two layers of bituminous hessian protected by 2 in. of concrete, above which the ballast and rails are laid. The total quantity of concrete in the slab is 360 cu. yd. The reinforcement in the top and bottom faces of the slab is 6-in. square-mesh steel fabric weighing 4¾ lb. per square yard. Along each of the outer edges of the fabric, a ⅝-in. mild steel bar is provided longitudinally. The slab is tied into the outer spandrel walls by new 1-in. tie-rods 6 ft. 6 in. long at 9-ft. centres. A nut bearing on a wall-plate 12-in. square is provided at the outer end; the inner end is plain (*Fig.* 2).



SECTION AT PIER

Fig. 2.—Transverse Section.

On completion of the slab and ballasting of one half of the viaduct, the single line is laid on this half, thereby allowing work to proceed in a similar sequence on the other half. Before the slab on the second half is constructed, however, the bag-wall is demolished.

Reconstruction of Quays at Le Havre.

PRESTRESSED CONCRETE DECKS AND PONTOON.

SOME unusual methods were used in the construction of the Saïgon-Plata quays at Le Havre to replace those damaged during the war. The new quays (*Fig.* 1), which will accommodate ships up to 24,000 tons displacement and will have a total length of 4000 ft. in a straight line, have prestressed concrete decks supported on cylindrical concrete piers at 27-ft. centres longitudinally and 30 ft. transversely. Each pier, which is 5 ft. diameter, is supported on a pile driven to a depth of 100 ft. to 140 ft. and formed by a French system called "Benoto". Holes of 5 ft. diameter are formed in the sea-bed by sinking by mechanically oscillated action steel tubes in several lengths welded to each other. As the tubes sink, the material in the interior of the tube is excavated by a clam-shell excavator (*Fig.* 3) known as a hammer-grab. A cage of reinforcement is inserted in each hole, which is then filled in stages with aggregate into which colloidal cement grout is pumped while the steel tube is being withdrawn with an oscillating motion. The part of the pier above the sea-bed is constructed with precast concrete rings filled with the same material as the piles.

The deck is constructed with prestressed concrete slabs each 82 ft. square and weighing about 1350 tons. The top of each slab is 7 in. thick and below this there are four transverse and three longitudinal ribs 1 ft. $7\frac{1}{2}$ in. thick and 11 ft. 6 in. deep. In each compartment formed by these ribs there are secondary transverse ribs to stiffen the slab. The slabs are cast four at a time on four beds, each pair of which is separated by a dredged basin. Each casting bed has a traversing gantry-crane (*Fig.* 4). The seating of the slab on a pier is at the intersection of two main ribs. The slabs are floated from the casting bed to the site and erected in accordance with the following procedure.

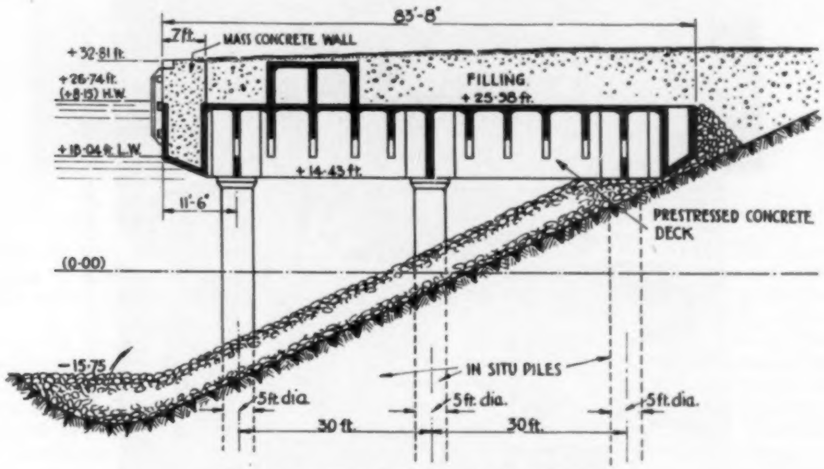


Fig. 1.—Cross Section of New Quays.

While the piers were being prepared a prestressed concrete pontoon 83 ft. by 110 ft. and weighing 1700 tons was constructed over the water on 63 timber piles (*Fig.* 2). The pontoon is 17 ft. deep and of cellular construction. The thickness of the cross walls varies from 3 in. to 8 in. The top and bottom slabs are only $4\frac{3}{4}$ in. thick. Each cell can be filled with water or emptied separately. The pontoon was launched (*Fig.* 5) in April, 1950. Most of the timber piles were first dynamited, and then the pontoon was pushed at one end by hydraulic jacks and

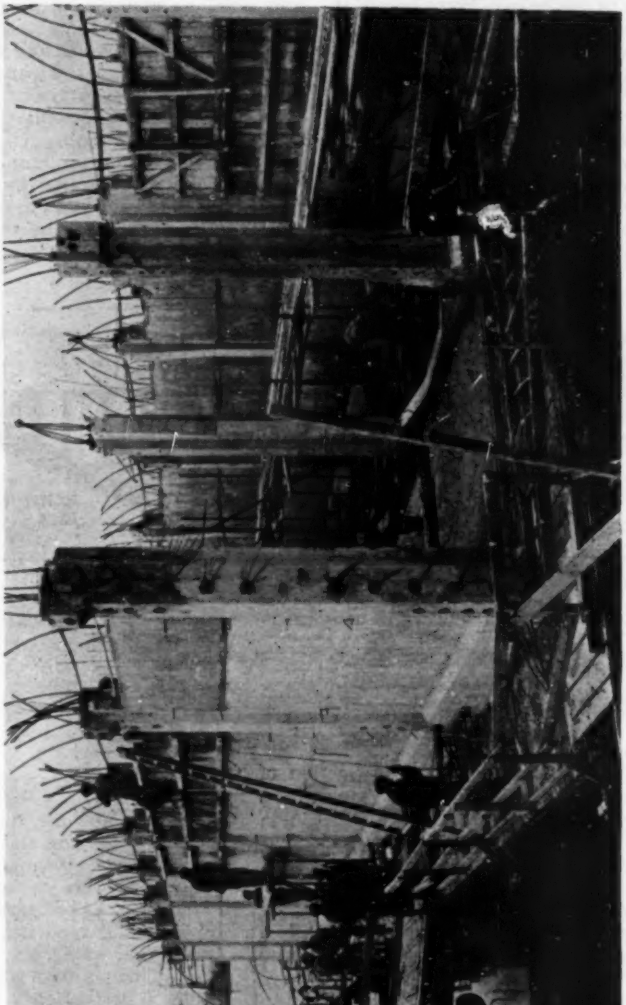


Fig. 2.—Pontoon Under Construction.

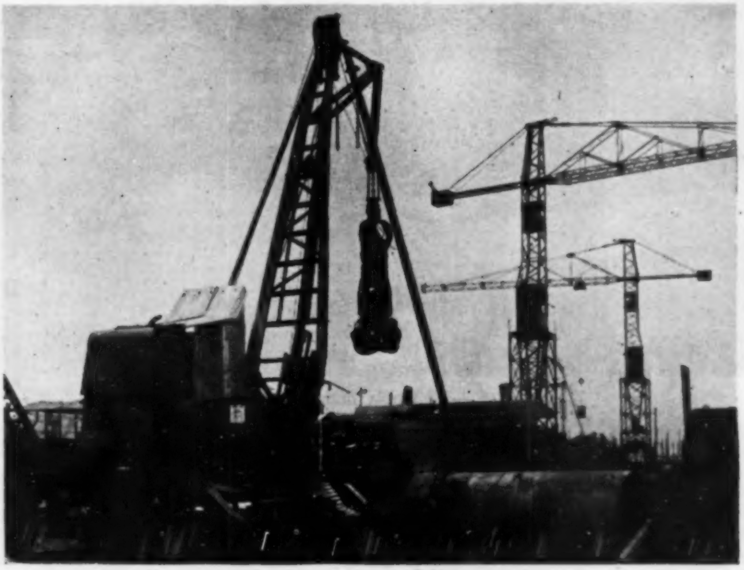


Fig. 3.—Piling Plant.

pulled at the other by two tugs. The pumps on the pontoon are in two water-tight steel superstructures.

The pontoon draws 6 ft. 6 in. of water when not loaded and 11 ft. 6 in. when loaded with one of the deck slabs. Twice a month at high tide, two of the deck slabs are ready for launching. The pontoon is brought into the basin between the casting beds and one of the deck slabs is placed on it by being slid sideways on lines of 4-in. diameter steel balls held in position between steel rails. The pontoon, bearing the deck slab, is moved out into deep water where it is sunk by admitting water into some of the cells, while the deck slab floats by virtue of the buoyancy imparted by the air trapped in the cells on its underside. The degree of buoyancy can be regulated by injecting compressed air below the slab. The slab is drawn over its permanent position by cables from hand-winches on the shore, and when in place it is lowered on to the

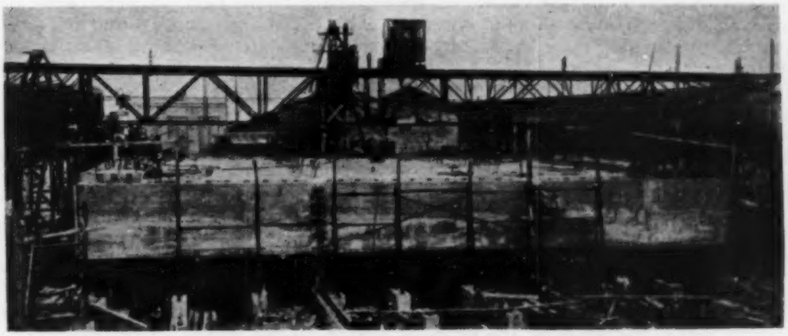


Fig. 4.—Casting Prestressed Concrete Deck Unit.

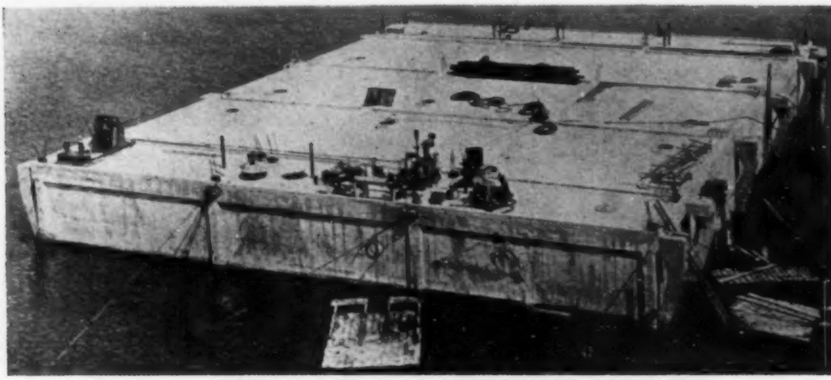


Fig. 5.—Pontoon after Launching.

piers by releasing the entrapped air. The first slab was floated into place in July, 1950, since when the quays are being constructed at the rate of 328 ft. monthly.

After releasing its load by sinking, the pontoon is brought to the surface and towed to the basin to bring out the next slab. When the pontoon has served its purpose on this site it is intended to be used as a floating dock.

The deck slabs and pontoon are prestressed by the Freyssinet method using cables of twelve or eighteen 0·2-in. diameter high-tensile steel wires around an open-coil spacer of 0·08-in. diameter mild steel wire. The cables, which are placed in the moulds before concreting, are prevented from bonding with the concrete by being encased in a sheath of thin strip steel, and are made and placed in the sheaths by a machine into one end of which the wires are fed and at the other the completed cable emerges.

The contractors are the Entreprises Campenon Bernard and the Société de Construction des Batignolles.

Defects in Road Foundations.

In "The Investigation of Road Foundation Failures" (Published by H.M. Stationery Office. Price 1*s*. 6*d*.; 40 cents. in U.S.A.) a procedure is described for determining the causes of the cracking and settlement of road surfaces when these defects arise from an unsatisfactory foundation. Trenches are cut in the road to expose the subgrade where the road is sound and where it is defective. The type and thickness of the courses comprising the road and the properties of the materials are determined. Tests are made to determine the type of soil, the variation of moisture content with depth is measured, and the position of the water-table and the occurrence of seeping are noted. The information obtained is useful in assessing the efficiency of the drainage and in determining the mode of entry of moisture into the subgrade. Tests of the subgrade to determine its stability include tests to determine the dry density, unconfined compressive strength, California bearing ratio, and modulus of subgrade reaction. Comparisons of the values obtained at the sound and defective sites indicate whether the condition of the road is due to a difference in the strength of the subgrade or of the road construction. The compressive strength and bearing ratio of the subgrade are then used to calculate the required thickness of the road, and comparison with the actual thickness may provide a further indication of the cause of the defects. Investigations of defective concrete and bituminous roads are described, most of which were on heavy clay subgrades of which the average moisture content was from 2 per cent. to 10 per cent. more than the plastic limit of the soil.

A North-Light "Shell" Roof.

THE accompanying illustrations show some of the stages in the construction of the roof of a factory at Hendon for Duple Motor Bodies, Ltd. The building (*Fig.* 1) is 200 ft. long and 100 ft. wide, and the clear height is 22 ft. The north-light roof is of shell construction, the spacing of the stiffeners being 50 ft. and the width of the curved slab 33 ft. 4 in.

The reinforced concrete columns each comprise three 18-in. square precast shafts. The central shaft, which supports the stiffener, is 24 ft. high and weighs 3·6 tons. The two outer shafts end at the underside of the prestressed

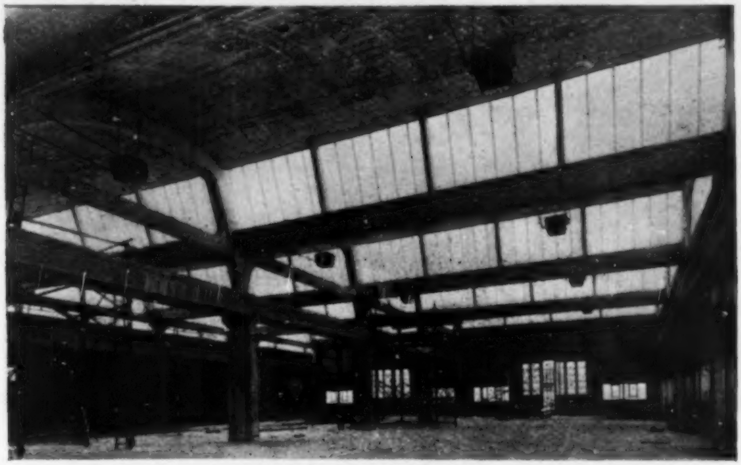


Fig. 1.—Interior View.

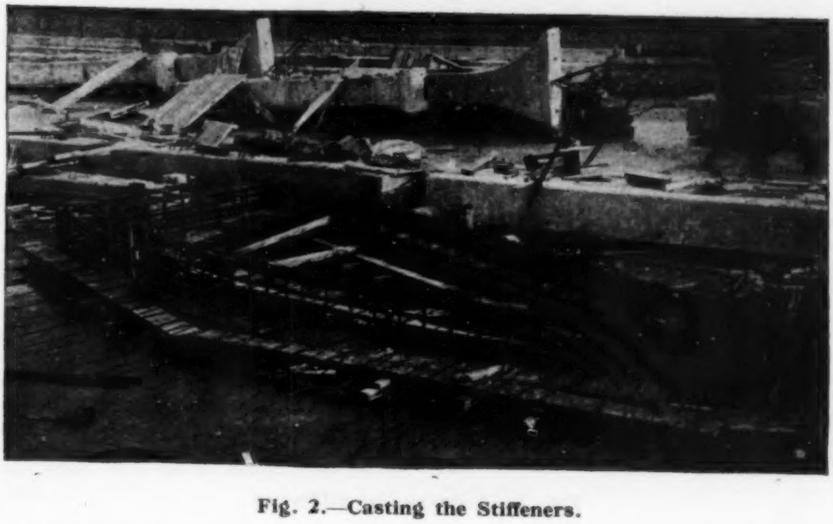


Fig. 2.—Casting the Stiffeners.

concrete crane-rail beams; these are 17 ft. high and each weighs 1·2 tons. The shafts were precast separately with ½-in. diameter bars protruding to form a bond with four connecting blocks of concrete cast in situ after the columns were erected. The lower end of each column is embedded in a cast-in-situ 1 : 2 : 4 concrete pile-cap supported on two 14-in. square piles 25 ft. long. Four reinforcement bars projecting from the splayed head of the central shaft tie into a block of concrete cast in situ after erection of the stiffeners. The proportions of the concrete in the columns and stiffeners is 1 : 2 : 4.

The reinforced concrete stiffeners, each

Fig. 3.—Casting the Stiffeners.

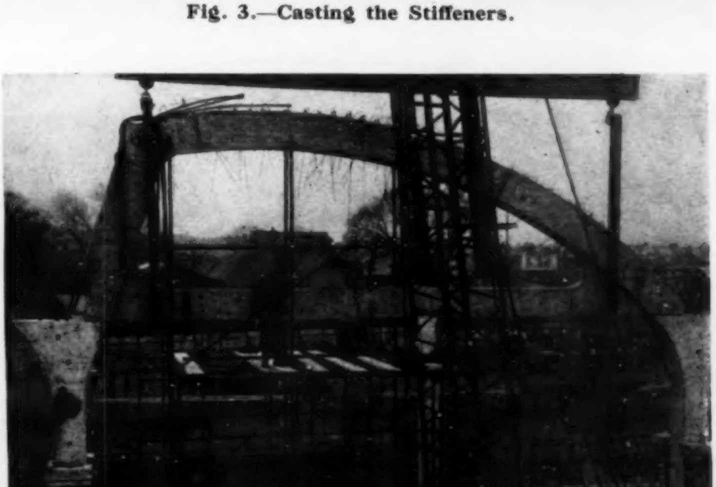


Fig. 4.—Erecting the Stiffeners.

of which weighs 10 tons, were cast on the ground floor (*Figs*. 2 and 3) and erected by a 12½-tons mobile crane (*Fig*. 4). Reinforcement projecting from the extrados is tied into the curved slab, and other projecting bars are embedded in the cast-in-situ valley gutters and the blocks on the column heads. The slab, which is 3 in. thick, the edge beams, and the valley gutters were cast in situ, and are of 1 : 1½ : 3 concrete.

The prestressed concrete crane-rail beams, each of which is 33 ft. long and weighs 5 tons, are designed to carry two 2-tons overhead cranes, and were made in a factory. The prestress was imposed, after the concrete had hardened, by the Freyssinet process. Each beam contains four parabolic cables of twelve 0·2-in. wires having a tensile strength of from 100 tons to 110 tons per square inch. After stretching, the cables were grouted with a 1 : 1 mixture of fine sand and cement. The concrete in the beams was made with rapid-hardening Portland cement; the crushing strength of cubes was 5400 lb. per square inch at the time of prestressing and 7000 lb. per square inch at 14 days.

The architects are Messrs. Welch & Lander, F./F.R.I.B.A. The consulting engineers are Messrs. C. W. Glover & Partners. The shell roof was designed by Mr. C. V. Blumfield, A.M.Inst.C.E. The contractors were Sir Robert McAlpine & Sons, Ltd. The prestressed beams were made by Messrs. C. H. Chaston & Co., Ltd.

Casting and Driving Collar-Piles.

For the foundation of a gasholder tank on a difficult site at Dumbarton, 1425 precast reinforced concrete main collar-piles and 575 precast reinforced concrete sheet-piles have been driven. The main piles (*Fig*. 1) are 25 ft. long and have enlarged sections near the head, square tapered shafts, and points without shoes.

Fig. 1.

A short length of 14-in. square shaft, which takes the blow from the hammer, extends above the enlarged section which is 2 ft. diameter. The precast circular collars are laid on the ground (*Fig*. 2) and the main piles are driven through them, the underside of the enlargement engaging the sloping sides of the opening in the collar and the collar being driven with the pile a short distance into the ground.

The sheet-piles are 25 ft. long, 13½ in. wide, and 6 in. thick, and are shaped at the sides to form a bird-mouth interlock.

Casting.

The sheet-piles were cast first and used as a bed upon which to cast the main piles. The casting-bed (A in *Fig. 3a*) for the sheet piles was 650 ft. long and 30 ft. wide and was formed by depositing a thin layer of concrete on ashes. A bed of this size enabled all the sheet-piles (B) to be cast with their widest face horizontal and all were left on the bed while the main piles (C) were cast. The bed accommodated 290 main piles, and these piles were therefore cast in five batches. *Fig. 3b* shows the method of casting the sheet-piles. The spaces (*E*) were blocked out while piles (D) were cast. When piles (D) had hardened, the separating moulds were removed and the vertical edges of (D) painted with bitumen. Reinforcement and concrete were then placed to form piles (E).

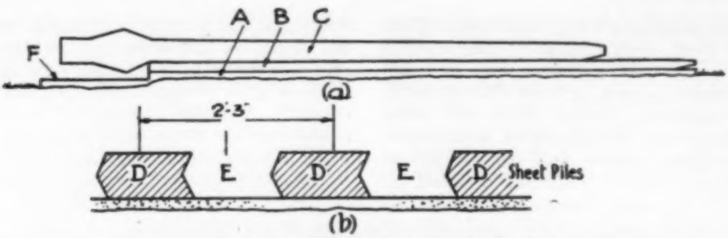


Fig. 3.—Method of Casting Piles.

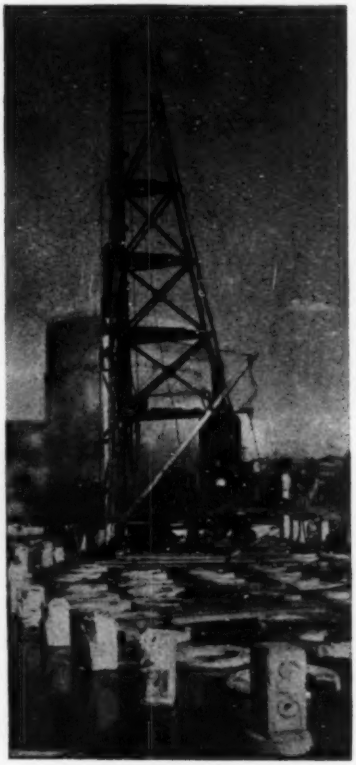


Fig. 2.

Steel moulds were used for the enlargement at the heads of the main piles, the moulds being carried in a cradle supported on a lower part (F) of the casting-bed. Shuttering for the tapered sides was provided by separating moulds placed between adjacent piles. The collars were cast in steel moulds.

The concrete had a water-cement ratio of 0·5 and was consolidated by tamping. It was mixed in two 14/10 mixers and taken to the casting-bed in concrete carts. Plates laid over the spaces between the piles enabled the carts to be brought into positions from which the concrete could be deposited directly into the moulds.

Driving.

The piles were taken to the piling-frames by a 2-tons crane on endless track. No difficulty was experienced in removing the piles from the casting-bed or in separating the sheet-piles. The main piles were driven by two frames (one of which is seen in *Fig.* 2) working in two shifts. Two-tons single-acting steam hammers were used. The sheet-piles were driven by a portable double-acting steam hammer assisted by jetting.

The work was carried out under the supervision of Mr. A. McFadyen, Engineer and Manager of the Dumbarton Gas Works. The consulting engineers are Messrs. F. A. Macdonald & Partners, and the contractors are Brydon Construction Co., Ltd.

Standard Dimensions of Buildings.

THE first report of the Committee of the British Standards Institution on Modular Co-ordination (B.S. 1708–1951; price 2*s.* 6*d.* from the Institution) recommends that horizontal dimensions of buildings should be any multiple of 40 in., and vertical dimensions should be certain multiples of 8 in. The report expresses the opinion that a small module, for example 4 in. as adopted in the U.S.A. and 10 cm. on the Continent, is not so useful as a larger dimension. Small modules lead to a multiplicity of standards and really standardise only small products. The advantages in manufacture and site work are only apparent, it is stated, if a larger module is adopted.

Repair of an Arch Bridge with Expanding Cement.

DURING the construction of a reinforced concrete road bridge (*Fig.* 1) at St. Julien in the south-west of France, settlement of one of the temporary piers in the river caused damage to the cellular arch and expanding cement was used in the repair work. The span of the arch is 328 ft., the rise-to-span ratio is about one-tenth, the width is 20 ft. 8 in., and the thickness varies from 4 ft. 11 in. to 7 ft. 6 in. The thickness of the walls of the three cells is 10 in. The abutments are on rock.

The settlement of the pier while the arch was being constructed in segments (*Fig.* 2) caused the arch to twist, and cracks (*Fig.* 3) occurred in the segment next to the left-hand abutment. The arch was tied temporarily so to be self-supporting in the event of further movement due to a possible flood, which did not, however, occur.

Due to the elastic settlement of the centering, the arch was partly supported on the abutments. At the left-hand abutment, the arch was supported only through the intrados slab, since the crack at the springing destroyed any support above the slab. To restore the arch to the original condition, it was proposed to exert a thrust *N* between the arch and the abutment. Two methods were considered, the first being to impose the thrust by means of jacks placed in a cavity cut in the arch ; while the jacks were in operation, the cavity would be filled with concrete, the jacks would be removed when the concrete had hardened, and the space they occupied also filled with concrete. The second method was to insert a segment of expanding-cement concrete, and this method was adopted because it would give a uniformly-distributed pressure, which would attain its maximum only after several days, instead of the isolated and rapid compressive actions of the jacks. The four operations of installing the jacks, concreting between the jacks, removal of the jacks, and the filling of the spaces which they occupied, was replaced by one operation, and the absence of mechanical devices reduced the possibility of unforeseen difficulties. The method was carried out as follows.

Fig. 1.—Bridge of St. Julien (showing old bridge in foreground).

The concrete of the arch was cut out at right-angles to the crack at the springing for the entire width of the arch and for a length of about 20 in. (*Fig.* 3). In the cut thus made, the segment was constructed with concrete containing 1000 lb. of expanding cement. The cement was made so that a pure paste in the free state would swell, upon wetting, 0·012 per cent. to 0·015 per cent. linearly. To ensure that the concrete was moistened sufficiently to cause the tendency to expand, holes (t), $1\frac{1}{4}$ in. diameter, were made (*Fig.* 3) by steel bars which were removed before the concrete set. A temporary mortar dam (m) retained a pool of water (E) on the segment (*Figs.* 3 and 4), which maintained the moistening for twelve days, which was the period required for the expansive force to be developed. Instruments recording the effects were attached at several points of the segment.

Cubes of the expanding-cement concrete made at the site had compressive

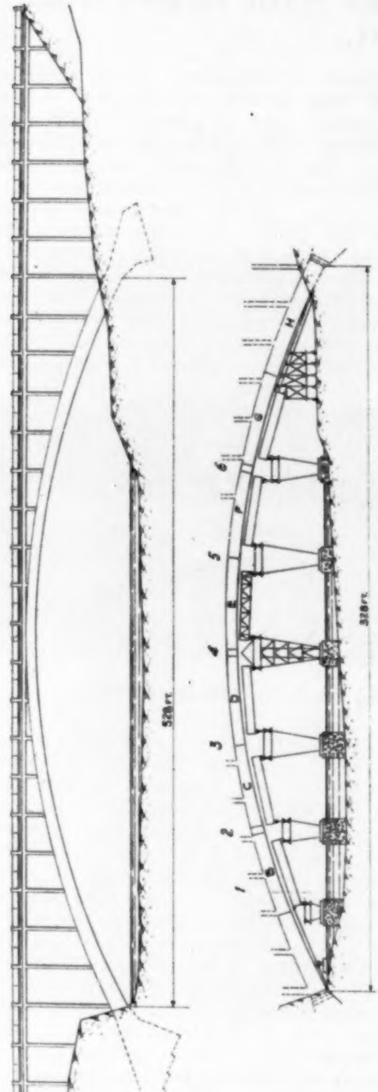

528 FT.
1
2
3
4
5
6

Fig. 2.

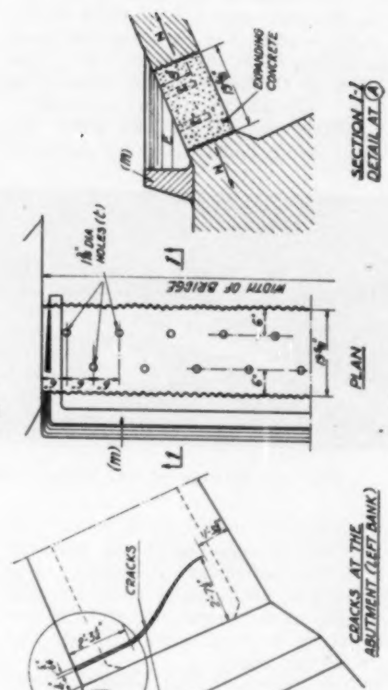

CRACKS
A
4'-7"
CRACKS AT THE ABUTMENT (LEFT BANK)
WIDTH OF BRIDGE
PLAN
(m)
EXPANDING CONCRETE
SECTION 1-1 DETAIL AT A

Fig. 3.

Fig. 4.

strengths of 6941 lb. to 7282 lb. per square inch at 90 days, compared with a calculated strength of 4580 lb. per square inch and the working compressive stress of 1280 lb. per square inch. The actual strengths (in lb. per square inch) at other ages were 4082 to 4409 at eight days and 5405 to 7823 at 28 days. The centering (*Fig.* 2) was removed thirteen days after the concreting of the segments.

The work was carried out by Etablissements Fourre et Rhodes to the design of Monsieur H. Lossier, who was a pioneer in the production of expanding cements.

Reinforced Aerated-Concrete Roof Slabs.

THE works and maintenance building at the new refinery of the Anglo-American Oil Co., Ltd., at Fawley, Hants, is a steel structure 800 ft. long and 180 ft. wide, and was erected in five months in 1950. The roof, which is of precast lightweight concrete slabs 8 ft. long, 16 in. wide, and 2¾ in. thick, is unusual because of the use of reinforced aerated concrete. The edges of the slabs are tongued and grooved, and a layer of steel fabric reinforcement weighing 4¾ lb. per square yard is placed about ½ in. from the top and bottom faces. The aerated concrete weighs about 75 lb. per cubic foot and has a compressive strength of about 1000 lb. per square inch. A typical test of the slabs is to place a load of 1 ton at the centre of a span of 8 ft. In one case this load produced a deflection of 6½ in. after three hours; when unloaded the residual deflection was ⅜ in. The slab was then turned over, and a load of 1 ton caused similar deflections. The safe uniformly-distributed load, without undue deflection, on 2¾-in. slabs is 200 lb. per square foot on a span of 8 ft.

Fig. 1.—Placing Roof Slabs in Position.

The concrete comprises clinker mixed with Portland cement in the proportions of 3 to 1 by volume, and a foaming agent. In addition to the water in the foaming agent, the concrete was mixed with 4½ to 5 gallons of water to 1 cu. ft. of cement. In cold weather rapid-hardening Portland cement to which calcium chloride had been mixed by the makers was used, and at other times the cement was a mixture of equal parts of rapid-hardening and ordinary Portland cement. The clinker was ⅜ in. to dust, and complied with the requirements of Class B material in British Standard No. 1165 (1947).

The slabs can be cut by a mechanical saw, and are nailable, rot-proof, and fire resistant. The coefficient of conductivity per inch of thickness is 2·25 B.T.U. per square foot per hour per deg. F., and the coefficient of thermal transmission is 0·48 for a 2¾-in. slab. The slabs are not considered to be impermeable, and are covered with four layers of felt saturated with asphalt and jointed with asphalt and surfaced with gravel.

The slabs were lifted on to the roof six at a time on an air-hoist. From the head of the hoist the slabs were moved into position on hand trucks (*Fig.* 1). After the slabs were laid, the joints were grouted level with the top surface. The contractors were Messrs. Foster Wheeler, Ltd., and the slabs were made by the Blokcrete Co., Ltd., at their concrete products works at Southampton.

Research on Reinforced Concrete in Sweden.

FOUR bulletins (1) published in 1950 by the Division of Building Statics and Structural Engineering of the Royal Institute of Technology, Stockholm, deal with tests and theories relating to common problems in reinforced concrete, and an abstract of each is given in the following.

Compressive Reinforcement in Beams.

A simplified method of designing concrete beams with compressive reinforcement is described in Bulletin No. 4. The method can be used, if the ordinary theory is used, for the calculation of the resistance with tensile reinforcement only. The compressive reinforcement is utilised with regard to the quality of the steel, which is not generally taken into account. The moment of resistance M is the sum of the moment of resistance M_p of the simply-reinforced section (in which the ratio of tensile reinforcement is p_t), and the moment of resistance ($M_s = c_s a_s A_c$) due to the compressive reinforcement and the corresponding amount of tensile reinforcement. The following formulæ apply:

$$M = M_p + c_s a_s A_c. \quad p_t = \frac{A_T - \frac{t_{yc}}{t_{yt}} A_c}{bd}.$$

A_T and A_c are the areas of the tensile and compressive reinforcement respectively; t_{yt} and t_{yc} are the yield-point stresses of the tensile and compressive reinforcement respectively; c_s is the permissible stress in the compressive reinforcement; a_s is the distance between the compressive and tensile reinforcement; and b and d are respectively the breadth and effective depth of the section.

Tests on concrete beams with compressive reinforcement, and having a maximum ratio of reinforcement of 2·5 per cent., show close agreement between the actual ultimate moments of resistance and those calculated by the proposed method. The ultimate load was independent of whether or not the beams were reinforced with stirrups; buckling of the compressive reinforcement occurred in both cases after the ultimate load had been reached and had decreased slightly. After buckling, the load-carrying capacity of beams without stirrups decreased more rapidly than that of beams with stirrups.

Slabs Spanning in Two Directions.

Rectangular reinforced concrete slabs supported on four sides are considered in Bulletin No. 5 from the point of view of the pattern of the cracks at failure of freely-supported square slabs with and without reinforcement at the corners, slabs clamped along two edges and differing in arrangement of reinforcement, and slabs clamped along two edges and submitted to loads for long periods. Tests show that the membrane effect produced by large deformations shortly before failure causes an increase of 10 per cent. to 15 per cent. in the ultimate load on freely-supported slabs. If the design is based on the theory of the lines of fracture, the factor of safety of slabs provided with two-way reinforcement is greater than that of slabs reinforced in one direction. Design bending moments of $\frac{wl^2}{20}$ for square slabs without corner reinforcement and $\frac{wl^2}{27 \cdot 4}$ for square slabs with corner reinforcement may give a sufficient factor of safety.

FLEXURAL RIGIDITY.—The decrease in the effective flexural rigidity, due to formation of cracks, occurs when the tensile stress in concrete (calculated from the bending moment based on the elastic theory) equals the tensile strength of concrete in bending. After the marked transition from the uncracked to the cracked condition, the greatest flexural rigidity was observed in slabs with corner reinforcement. The decrease in flexural rigidity after cracking is more noticeable in strips of slabs, indicating that the slab is no longer isotropic. Calculation of the flexural rigidity after cracking is therefore difficult because the torsional and flexural rigidities with various twisting and bending moments are not known. When the load causing cracking was exceeded, the flexural rigidity of the slabs decreased so much that the deformations were greater

(1) No. 4. "Concrete Beams with Compression Reinforcement." By A. Johnson.
No. 5. "Concrete Slabs Reinforced in Two Directions." By H. Nylander.
No. 6. "Calculation of Deformations in Reinforced Concrete Structures after Formation of Cracks."
No. 7.—"Transfer of Moments and Deformations in Concrete Beams Submitted to Long-time Loads." By A. Johnson.

The bulletins are in the Swedish language with a summary in English, and are obtainable from the Institute. (No price stated.)

than is allowable in structures. The difference between the observed and the calculated moments of resistance was not so great for the clamped slabs as for the freely-supported slabs, which may indicate that the membrane stresses in clamped slabs are less important than in freely-supported slabs. The effective flexural rigidities of all types of slabs tested were the same until diagonal cracks formed in the bottom, when a great decrease in flexural rigidity was observed. The formation of cracks over the supports had little effect on the flexural rigidity before cracks occurred in the bottom. The greatest flexural rigidity was exhibited by slabs with the greatest amount of reinforcement at the supports, and the transition from one condition to the other also was noticeable when the tensile stress in the concrete (calculated from the maximum positive bending moment based on the elastic theory) was nearly equal to the tensile strength of concrete in bending.

Frequent loading and unloading, and loading for long periods, had no influence on the ultimate resistance, the load which caused cracks to spread over the entire slab being also the critical load as regards the increase in deflection. In comparison with loading for a short period, the load was not much reduced on account of the slab having been previously submitted to frequent loading and unloading and loading for long periods. The increase of deflection with time was particularly noticeable after the formation of the first cracks in the bottom of the slab.

Effect of Shrinkage.—Tests on vibrated concrete beams showed that when shrinkage is not uniform throughout the depth the deformations and distribution of the bending moments are affected. Excessive or insufficient vibration results in shrinkage that is so variable that it cannot be disregarded in estimating the structural action of a slab. Calculations by the elastic theory show that non-uniform shrinkage and the prevention of shrinkage by reinforcement cause increased deflection of a freely-supported slab, and the increase cannot be neglected in determining the flexural rigidity. If a slab is clamped along one or two edges, the ratio of the deflection caused by shrinkage to the deflection produced by the load is smaller than in the case of the freely-supported slab, but at the supports the bending moments and stresses due to shrinkage are about the same as, and may be greater than, the bending moments and stresses due to the maximum load. Consequently, cracking at the supports may occur causing the flexural rigidity of the slab to decrease near the supports, and the deflection becomes still greater. If a slab is clamped along four edges, the effect of shrinkage is less dangerous, the bending moments and stresses at the supports being increased slightly by variable shrinkage; the stresses due to the restraint of shrinkage by the reinforcement are hardly increased.

Transfer of Moments due to Formation of Cracks.—It is supposed that the moment-distribution at failure of a slab with two-way reinforcement largely adapts itself to the arrangement of the reinforcement. With small loads, the actual moment-distribution differs from that determined by the elastic theory, chiefly owing to shrinking and cracking. Tests showed that due to cracking the bending moments at the supports are smaller than the moments computed by the elastic theory. The moment-distribution was not influenced by loading for long periods, but was affected by the formation of cracks at the time the load was applied and during the following few days. Alternate loading did not cause any notable change in the moment-distribution.

Arrangement of Reinforcement.—The manner in which the bars are terminated influences the amount of reinforcement. It is essential that the bars be securely anchored. Tests show that half the bars in an ordinary freely-supported slab can be terminated according to the moment-curve and the remainder should extend to the point at which there is no bending moment. A similar rule applies to regions of positive bending moment in the direction of the greatest positive moment in a continuous slab. The positive bending moment in the other direction may increase near the support more rapidly than in the direction of the greatest positive bending moment, but may vary less towards the middle of the span. In the direction of least bending moment the bars should end at not more than the same distance from the point of no moment as in the direction of maximum positive bending moment. Anchorage of the bars over the supports is probably provided by extending the

bars beyond the point of no moment a distance equal to half the required bond length, the ends of the bars being staggered as in ordinary design. Tests showed that the flexural rigidity of slabs in which half the bars terminate according to the moment-curve is not appreciably less than if all bars extend to the point of no moment.

DESIGN.—In view of the wide variations in the properties of concrete, the object of design, it is stated, should be to provide safety against harmful fractures and deformation. Safety against fracture can be assured if the design moments are determined on the basis of the fracture stage at which the distribution of the bending moments largely conforms to the arrangement of the reinforcement. Initial stresses due to shrinking are of no importance at the fracture stage. The simplest method of determining the magnitude of the design moments is the theory based on the lines of fracture. For slabs simply supported it is suggested that the bending-moment coefficients determined by this theory should be adjusted to allow for the membrane effect, the reduction being $10(2 - r)$ per cent., where r is the ratio of the length of the longer side to the shorter side.

In considering safety against large deformations, the sum of the maximum deflection (based on the elastic theory with creep taken into account in evaluating the elastic modulus) and the deflection due to shrinking should not exceed a proportion of the least lateral dimension of the slab. Also, cracking should not be so extensive that there is a considerable reduction in the rigidity of the slab, a requirement which is fulfilled if the tensile stress due to the maximum positive bending moment is less than the tensile bending strength of the concrete. A rule limiting the magnitude of the deflection is desirable if moderate deflections may have objectionable effects. A proposed rule is that the ratio of the maximum deflection to the length of the shorter span b should not exceed 0·002. A rule embodying this ratio and allowing for elastic deflection, deflection due to creep and shrinking, and for the fact that the effective rigidity must not be appreciably reduced by cracks, is given by

$$\frac{b}{H} < \sqrt{\frac{c_t}{6w(k + 0{\cdot}01)}}$$

in which H is the thickness of the slab, c_t the modulus of rupture, w the intensity of total load, and k the coefficient for the maximum bending moment depending on the conditions of restraint at the edges. (Values of $k = n_{max.}$ are given in the bulletin.)

Deformation of Cracked Structures.

In Bulletin No. 6, the deformations of reinforced concrete structures after cracking are calculated on the basis of the properties of the materials, the amount of reinforcement, and the effect of tension in concrete. Reinforced concrete prisms subjected to tension are considered as the fundamental type, the total strain ε of a prism being given by

$$\varepsilon = \frac{1}{E_s}\left(t - \frac{c_t}{2p}\right)$$

that is ε is dependent on the ratio p of reinforcement and the tensile strength c_t of the concrete, and is independent of the size of the bars and the strength of bond between concrete and steel. The strain of the reinforcement in a beam is the deformation, ignoring the tensile strength of the concrete, reduced by a constant amount, dependent only on c_t and p. Calculations are in good agreement with the results of tests. The deformations of concrete in compression and in tension serve as a guide in determining the flexural rigidity E_eI, and deflection, which can be calculated first by ignoring the tensile strength of the concrete and then reduced by a constant amount to allow for this tensile strength. Loading and unloading decreases the effect of tension in the concrete, with the result that deformations are more nearly equal to the calculated value without the reduction. It is therefore difficult to predetermine accurately the shape of the deformation curve after cracking, but it is possible to draw two limiting curves for the deformations.

It is shown that reinforced concrete slabs almost completely lose their torsional rigidity after the development of cracks, and act as beam grids without torsional restraint, thus accounting for the great increase of the deflection of slabs after the formation of cracks.

Effect of Long-time Loading.

From the tests on continuous beams under loading of long duration described

in Bulletin No. 7, the author draws the following conclusions.

At ordinary stresses, plastic flow does not seem to influence the distribution of the bending moments. Shrinkage increases generally the moment at the support whether the concrete is cracked or not. Cracking usually causes transfer of moments depending on the ratio of the amounts of reinforcement at the support and at midspan, an increase in the reinforcement at the support causing an increase in the moment there. The change in the moment at the support caused by shrinkage is generally greater than the change due to cracking of the concrete.

The deflection is increased by plastic flow, cracking, and shrinkage. The ratio of the amounts of reinforcement at the support and at midspan has little influence on the deflection if the yield-point stress is not exceeded whether or not the concrete is cracked. Consequently, an amount of reinforcement differing largely from that calculated in accordance with the elastic theory may not cause extra deflection.

Revised Requirements for Bond in the American Code.

REVISIONS made recently to the code for reinforced concrete of the American Concrete Institute decrease the allowable bond stress on plain bars and older types of deformed bars and increase the allowable bond stress on new types of deformed bars. Top bars having more than 12 in. of concrete under them are assigned lower bond stresses than bars in other positions. All plain bars must now be hooked. Some of the new clauses are given in the following.

The definition of a deformed bar is "a reinforcing bar conforming to the Standard Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement (A.S.T.M. Designation : A 305). Bars not conforming to these specifications shall be classed as plain bars. Wire mesh with welded intersections not farther apart than 6 in. in the direction of the principal reinforcement and with cross wires not smaller than No. 10 gauge may be rated as a deformed bar."

"In simple beams, or at the freely supported end of continuous beams, at least one-third the required positive reinforcement shall extend along the same face of the beam into the support a distance of 6 in."

"Plain bars in tension shall terminate in standard hooks, except that hooks shall not be required on the positive reinforcement at interior supports of continuous members."

The allowable bond stresses are now :

	Allowable stress	*Max. stress (lb. per sq. in.).*
Deformed bars		
Top bars	$0{\cdot}07f'_c$	245
In 2-way footings (except top bars)	$0{\cdot}08f'_c$	280
All others.	$0{\cdot}10f'_c$	350
Plain bars (must be hooked)		
Top bars	$0{\cdot}03f'_c$	105
In 2-way footings (except top bars)	$0{\cdot}036f'_c$	126
All others.	$0{\cdot}045f'_c$	158

The stress f'_c is the compressive strength of 6-in. cylinders at 28 days.

DATA FOR PRICING REINFORCED CONCRETE.

Materials.

(*Delivered in London area.*)

AGGREGATES (per cu. yd.).—Washed sand, 17*s*. Clean shingle: ¾ in., 15*s*. 2*d*.; ⅜ in., 17*s*. 6*d*. Thames ballast, 15*s*. 9*d*. Broken brick, ¾ in., 17*s*. 6*d*.

CEMENT (per ton, delivered at Charing Cross).—

Portland cement, 6 tons and upwards, 85*s*. 6*d*. 1 ton to 6 tons, 90*s*. 6*d*. Paper bags and non-returnable jute sacks included.

Rapid-hardening Portland, 6*s*. above ordinary Portland.

"417," 110*s*. in 6-ton loads; paper bags included.

Aquacrete, 31*s*. 6*d*. above ordinary Portland; paper bags included.

Colorcrete (buff, red, and khaki), in 6-ton loads, 127*s*.; paper bags included.

Snowcrete, £12 3*s*., inc. paper bags.

"Super-Cement," 31*s*. 6*d*. per ton above ordinary Portland cement; bags included.

High-alumina cement, 1 ton and upwards, 247*s*. per ton; paper bags 13*s*. 6*d*. per ton extra.

Snowcem paint, 56*s*. per cwt. inc. containers.

SHUTTERING.—For prices of timber, refer to S.R. & O., 1949, No. 1079 (price 1*s*. 1*d*.) and No. 94 (price 5*d*.) issued by H.M. Stationery Office.

REINFORCEMENT.—Mild steel round bars (per cwt.): 11/16 in. to 2⅞ in., 28*s*. 2*d*. 7/16 in. to ⅝ in., 29*s*. ⅜ in., 29*s*. 8*d*. ¼ in., 31*s*.

Materials and Labour.

(*Contracts up to £5000. Inc. 10 per cent. profit.*)

PORTLAND CEMENT CONCRETE, 1 : 2 : 4.—

Foundations, 2*s*. 2*d*. per cu. ft. Columns, 2*s*. 5*d*. per cu. ft. Beams, 2*s*. 5*d*. per cu. ft. Floor slabs 4 in. thick, 6*s*. 10*d*. per sq. yd.; Do., 5 in., 8*s*. 7*d*.; Do., 6 in., 10*s*. 3*d*.; Do., 7 in., 12*s*. Walls 6 in. thick, 10*s*. 3*d*. per sq. yd. Add for hoisting 3*s*. 6*d*. per cu. yd. above ground floor level. Add for rapid-hardening Portland cement 2*s*. per cu. yd.

REINFORCEMENT.—Mild steel round bars, including cutting, bending, fixing, and wire (per cwt.)—¼ in. to ⅜ in., 48*s*. 6*d*. 7/16 in. to ⅝ in., 43*s*. 6*d*. 11/16 in. to 2⅞ in., 42*s*.

SHUTTERING AND SUPPORTS.—

Walls, 155*s*. per square.

Floors (average 10 ft. high), 160*s*. per square. In small quantities, 1*s*. 8*d*. per sq. ft.

Columns, average 18 in. by 18 in. (per sq. ft.), 1*s*. 8*d*.; in narrow widths, 2*s*.

Beam sides and soffits, average 9 in. by 12 in. (per sq. ft.), 2*s*.; in narrow widths, 2*s*. 2*d*.

Raking, cutting, and waste, 5*d*. per lin. ft.

Labour on splays, 3*d*. per lin. ft.

Small fillets to form chamfers, 6*d*. per lin. ft.

Wages.

The rates of wages on which the above prices are based are: Carpenters and joiners, 3*s*. per hour (carpenters 2*d*. a day tool money); Labourers, 2*s*. 6*d*.; Men on mixers and hoists, 2*s*. 7*d*.; Bar-benders, 2*s*. 8*d*.

Competition.

A COMPETITION for ideas for improving the efficiency of the processes used in building and civil engineering is being held by Acrow (Engineers) Ltd. The prizes total £1000. The closing date for entries is July 2. Details may be had from the promoters at South Wharf, Paddington, London, W.2.

Partnership.

Mr. C. B. Brown, B.A.I., consulting engineer, of 16 Gloucester Place, London, W.1, has been joined in partnership by Dr. T. P. O'Sullivan, Ph.D., B.Sc.(Lond.). The practice is now known as Brown, O'Sullivan & Partners.

Change of Address.

THE address of Sealocrete Products, Ltd., is now Atlantic Works, Hythe Road, London, N.W.10. Telephone: Ladbroke 0015; Telegraphic Address: Exploiture, London.

MISCELLANEOUS ADVERTISEMENTS.

Situations Wanted, 3d. a word: minimum 7s. 6d. Situations Vacant, 4d. a word: minimum 10s. Other miscellaneous advertisements, 4d. a word: 10s. minimum. Box number 1s. extra.

Advertisements must reach this office by the 23rd of the month preceding publication.

SITUATIONS VACANT.

SITUATIONS VACANT. The Trussed Concrete Steel Co., Ltd., invite applications for the following progressive positions on their permanent staff. (A) Civil or structural engineers with extensive experience in design and/or construction of reinforced concrete structures. Must have ability to carry responsibility. (B) Detailers with sound experience. (C) Draughtsmen with some knowledge of the subject. All positions are progressive and pensionable. Vacancies (B) and (C) occur in London and Manchester, and (A) in London only. If you are interested will you please let us have full details of your experience to TRUSCON HOUSE, 35/41, Lower Marsh, London, S.E.1.

SITUATION VACANT. Civil and structural engineer required by the Cement and Concrete Association, age about 35–40 years, with wide experience of reinforced concrete design and construction and ability to express technical ideas in a clear and concise manner. The work includes the interpretation of technical information on concrete construction, advising on the use of concrete, and translating the results of research work to practice. The post is progressive and pensionable. Commencing salary according to age and experience. Write, giving particulars, to 52 Grosvenor Gardens, London, S.W.1.

SITUATION VACANT. Reinforced concrete detailer required for permanent staff, Westminster. Salary £500–£700, according to experience. Applications, which should state age, details of experience, and salary required, to Box 2449, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Engineer-designer required in the Glasgow office of a well-known firm of reinforced concrete engineers. Considerable experience in reinforced concrete design is an essential qualification. Also required junior designers and draughtsmen-detailers with sound experience in reinforced concrete drawing office work. The posts offer long-term employment to suitable applicants. Superannuation and profit-sharing schemes are in operation. Write, giving full particulars and past experience, to Box C.E. 590, at 191, Gresham House, London, E.C.2.

SITUATION VACANT. Consulting engineers, Westminster, require civil engineer as salaried partner, age between 30 and 45, in Karachi, Pakistan. Salary £1,400 to £1,800 per annum according to experience, plus bonus guaranteed at £200 p.a. minimum. Applicants should have a public school education and possess the A.M.I.C.E. qualifications or equivalent University degree, and should be experienced in public works and reinforced concrete, structural steelwork, etc., either with Contractors or consulting engineers. Three months' leave with free passage home every two years. Applications giving full details of education and experience to Box 2453, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Contracts manager required, specialising in reinforced concrete work, particularly in reinforced concrete frames, multi-story buildings, reservoirs, silos, etc. Good bonus on results. Permanent and pensionable position. House available if required. Write in first instance, stating age, salary required, and experience, to WATES, LTD., 1258 London Road, Norbury, London, S.W.16.

SITUATION VACANT. Manager required for modern precast concrete factory, Staines district, employing 80 men. Annual turnover exceeding £100,000, top quality products. Must be capable of directing existing staff, good salesman and negotiator, able to discuss projects with architects and engineers. Must have held similar position before. House available if required. Top salary paid, together with bonus on results. Permanent and pensionable position for the right man. Please write in first instance, stating age, salary required, and experience, to WATES, LTD., 1258 London Road, Norbury, London, S.W.16.

SITUATIONS VACANT. Howard Humphreys & Sons require immediately the following staff for sewerage and water supply schemes. (1) Draughtsmen with not less than five years' experience and (2) draughtsmen with three years' experience in one or both of these branches of civil engineering. Also (3) draughtsmen with not less than four years' experience in reinforced concrete detailing. Full particulars of training and experience, with dates, age, and salary required, in confidence, to 191 Victoria Street, London, S.W.1.

SITUATION VACANT. Civil engineer agent required to take charge of contracts in reinforced concrete involving barrel vault roof construction. The successful applicant will be based in the Greater London area, controlling contracts within a 40–50 mile radius. Only applicants who have the necessary ability and initiative to control and develop an expanding business, with considerable scope for progress and advancement within the company, need apply. Box 2455, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Old-established firm of consulting civil engineers has a number of vacancies for civil engineering assistants and reinforced concrete designers for general building work in connection with generating stations and a variety of industrial construction. Positions are of a permanent character, and opportunities for individual ability and initiative will be provided. A full-scale superannuation scheme is in operation. Salaries will in the first place be in accordance with experience and qualifications, but will be under constant review for adjustment in accordance with proved ability. The majority of applicants will be interviewed, when full details of the character of the work will be explained. Travelling expenses, etc., for interviews will be refunded. Applicants are asked to send full details of experience, etc., and approximate salary required, to C. S. ALLOTT & SON, North Parade, St. Mary's Parsonage, Manchester, 3.

SITUATIONS VACANT. Consulting engineers (structural), Westminster and Liverpool, are shortly opening a large new country office at Haslemere, Surrey, where the following vacancies are available on permanent staff. 2 Senior reinforced concrete detailers, commencing salary £650 to £750 p.a. 3 Junior reinforced concrete detailers, commencing salary £350 to £500 p.a. 5 Learners, starting salary £150 p.a. The office is within 5 minutes' easy walk of Haslemere main line station, and applications will be considered only from staff who live within easy daily travelling distance from Haslemere. Pension scheme. Applications, which will be treated as confidential, stating age, experience and salary required, to Box No. 2456, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Draughtsmen. Internationally known English company with drawing offices in Regent Street, London, require now a number of draughtsmen experienced in one of the following: (a) structural concrete (seniors with design knowledge), (b) electrical layouts, (c) piping of oil refinery or chemical or boiler plants or in underground drainage, (d) pressure vessels (tower and tank). Applicants with experience in other fields of draughtsmanship will be considered for conversion training. Generous salary will be given according to qualifications and experience. Good prospects of early promotion are offered to men of ability in this company, in a rapidly expanding industry. Good conditions of service, including pension scheme, sickness benefits and five-day week. Sports and social club. Special leave and pay for Territorials and "Z" men. Successful applicants will work in the London area and those selected for interview will be re-imbursed their travelling expenses. Replies, in strict confidence, and giving full details of education and employment history and any service record, to THE LUMMUS CO., LTD., Imperial House, 80 Regent Street, London, W.1.

SITUATION VACANT. Keen designer-detailer required by well-known company in South Lancashire. Must be capable of detailing large reinforced concrete structures working from designer's notes and sketches, and to undertake simple design. Good salary paid, staff bonus, and superannuation schemes available, and opportunity given to inspect work during construction. Full particulars to Box 2458, CONCRETE AND CONSTRUCTIONAL ENGINEERING, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Experienced concrete designers and detailers wanted for Southern Rhodesia. Candidates should have had at least five years' experience of competitive designing. Free passages. Salary according to experience and qualifications. Write details in confidence to Box 66/3, c/o 95 Bishopsgate, London, E.C.2.

(Continued on page lx.)

MISCELLANEOUS ADVERTISEMENTS.

(*Continued from page lix.*)

SITUATION VACANT. Architectural draughtsman required for the engineering department of a large metallurgical works at Rainham, Essex. Applicant should be age 21 to 25 years, should have a sound knowledge of building construction and prices, and should be capable of preparing working drawings with a minimum of supervision. The successful applicant will obtain experience in civil engineering work and will be given opportunities to learn the practical side of structural design. Please state age, experience, and salary. Box 226, c/o Dawson's, 129 Cannon Street, London, E.C.4.

SITUATIONS VACANT. Experienced designers required for East Africa by long-established company of reinforced concrete specialists employed primarily on design of commercial structures. Unmarried men preferred. 4½-year contract with paid passages. Salary commensurate with experience and qualifications. Write Box 67/3, c/o 95 Bishopsgate, London, E.C.2.

SITUATION VACANT. Reinforced concrete designer and detailer wanted for London office of professional engineers. Position offers exceptional opportunity for all-round experience. Good prospects of promotion. Box 2460, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION VACANT. Midland precast concrete works require assistant, 24–30, with initiative and drive, to take charge of section of production. Previous knowledge preferable, but not essential. Good prospects of advancement. Accommodation may be available. Staff pension scheme. Box 2461, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATIONS VACANT. Reinforced concrete detailers required. Apply, with details of experience, etc., to J. L. Kier & Co., Ltd., Civil Engineering Contractors, 7 Lygon Place, London, S.W.1.

SITUATION VACANT. Draughtsman with at least three years' experience in reinforced concrete detailing required immediately. Apply in writing to Peter Lind & Co., Ltd., Stratton House, Piccadilly, London, W.1, stating age, experience, and salary required.

SITUATIONS VACANT. Reinforced concrete designers and detailers required in W.1. office. Five-day week, canteen. Write, stating experience, to "Twisteel" Reinforcement, Ltd., 43 Upper Grosvenor Street, London, W.1.

SITUATION VACANT. Firm of consulting and manufacturing engineers requires a versatile reinforced concrete draughtsman with a knowledge of general building construction to work at Dagenham. Excellent conditions with permanency. Box 2462, Concrete and Constructional Engineering, 14 Dartmouth Street, London, W.1.

SITUATIONS VACANT. Civil Engineering Assistants (Unestablished). Applications are invited from British-born subjects to fill the above vacancies with the Ministry of Works at sites in various parts of the country but mainly in the south of England. Hostel accommodation is available at most sites. Candidates should be capable of reading building and civil engineering drawings and have knowledge of use of level and theodolite for setting-out and checking. Duties should prove of considerable value to men studying for civil engineering qualifications and needing site experience. Salary scale (Provincial) is £283 (age 21) to £495 per annum. Starting salaries would be assessed according to age, training, and experience, but would not normally exceed £420 per annum at age 28 and above. Written applications giving date of birth, education, full details of qualifications, and experience of posts held (including dates) should be addressed to Appointments Officer, Ministry of Labour and National Service, 1–6 Tavistock Square, London, W.C.1, quoting reference number KJ94, within 14 days of appearance of this advertisement. In no circumstances should original testimonials be forwarded. Only candidates selected for interview will be advised.

SITUATIONS WANTED.

PROFESSIONAL SERVICES. Two chartered engineers with 20 years' experience in the design of reinforced concrete and structural steelwork are prepared to complete designs, working drawings, bills of quantities, and bending schedules. Box 2457, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

SITUATION WANTED. Reinforced concrete designer with experience is able to undertake part-time work at home for consulting engineers. Design and detailing of all types of reinforced concrete structures. Write, giving your requirements, to Box 2459, Concrete and Constructional Engineering, 14 Dartmouth Street, London, S.W.1.

FOR SALE.

FOR SALE. Offers invited for "V" type trestle pier approximately 150 ft. long × 15 ft. high × 15 ft. wide. Components can be inspected at site. Apply to Robert M. Douglas (Contractors), Ltd., Trostre Works, Llanelly, Carms.

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PUBLIC NOTICE.

PNEUMOCONIOSIS.

The Minister of National Insurance has referred to the Industrial Injuries Advisory Council for further consideration the question of the method of prescribing pneumoconiosis under the National Insurance (Industrial Injuries) Act, 1946, i.e. how the classes of insured persons eligible for benefit for the disease should be defined. The disease is at present prescribed by reference to a schedule of occupations which are known to give rise to a risk of the disease. These occupations are set out in Part II of the First Schedule to the National Insurance (Industrial Injuries) (Prescribed Diseases) Regulations, 1948 (S.I. 1948, No. 1371).

The Council propose to review this method of prescription and to consider possible alternatives, e.g. prescription generally for all insured persons, or by reference to occupations involving exposure to concentrations of specified dusts. The Council may also re-consider the definition of pneumoconiosis which for the purposes of the Act means "fibrosis of the lungs due to silica dust, asbestos dust or other dust, and includes the condition of the lungs known as dust-reticulation".

The Council are prepared to receive evidence from any persons or bodies interested, who should communicate with their Secretary, Mr. S. E. Waldron, O.B.E., Ministry of National Insurance, 30 Euston Square, London, N.W.1, as soon as possible, and in any event not later than 1st May, 1951. An explanatory memorandum on the subject will be supplied on request.

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